Comments on Cape Cod Subsurface Gravel Wetland System Design

April 2, 2015

Bruce Jacobs
WaterVision LLC

Re: USEPA LID Implementation Project Design Review for Chatham and Barnstable Subsurface Gravel Wetland System Conceptual Designs

Dear Mr. Jacobs:

Thank you for the opportunity to provide design review of the Chatham and Barnstable Subsurface Gravel Wetland System Conceptual Designs. I hope you find the comments listed below informative and instructive.

Documents used for review

95% Project Design Plans

95% Design Calculations

Barnstable Model Outputs 3-26-15

Chatham Model Outputs 3-26-15

General Comments

Additional effort is needed with respect to both the Barnstable and the Chatham Subsurface Gravel Wetland (SGW) system designs.

General Recommendations

An intermediate layer of a graded aggregate filter (i.e., pea gravel) is needed to prevent the wetland soil from migrating down into the crushed-stone (gravel) sub-layer. This is to prevent migration of the finer setting bed (wetland soil) into the coarse sublayer. Material compatibility should be evaluated using FHWA criteria which is the criteria employed in the online UNHSC Subsurface Gravel Wetland specification (see Ferguson, 2005):

Criteria 1: $D_{15,\,COARSE\;\;SUBLAYER} \leq 5\,x\,D_{85,\,SETTING\;\;BED}$

Criteria 2: $D_{50,\,COARSE\;\;SUBLAYER} \leq 25\,x\,D_{50,\;\;SETTING\;\;BED}$

Hydraulic Inlet

The diversion wall for the inlet control structure needs clarification. For the Oyster pond BMP the elevation of the diversion wall is 15.33’ and the invert of the inlet pipe is 14.89, this indicates that once the 10” inlet pipe is 5.28” full the system will revert to bypass. This would mean that the dmax for the SGW system is 5.28” above the wetland soil surface. This has implications for all design elevations particularly with respect to the above ground storage volume of the system.

It is unclear why the elevation of the diversion wall wouldn't be 17' which would legitimize much of the design calculations.

A 3" diameter weep hole in the diversion wall has the potential to route small flows through the bypass prematurely. It is unclear as to why this is necessary as the system will be controlled by the outlet orifice and will drain to the inlet invert.

The nature of the inlet configuration will make monitoring extremely difficult. An upstream influent monitoring location should be established beyond the backwater influence of the system.

It is unclear how the solid 6" underdrain works in the proposed system.

The hydraulic outlet structure provide for high flow bypass with 7.8" of freeboard. This seems like a lot of freeboard considering the reduce storm depth treated and the inclusion of the upstream diversion wall. This secondary hydraulic outlet will also affect the amount of above ground storage volume in the basin, we typically provide a minimum of 2" of freeboard. The inclusion of a tertiary spillway (elevation unknown) creates plenty of redundancy for high flow system bypass.

The design storage calculations as well as the WQV:ISR ratios need to be recalculated to reflect the previous comments. For consistency the WQV should remain 20,742 cubic feet. It is understood that the system is undersized however to date the definition of the WQV is standardized by state and should remain so. Just because the design volume of the system is 0.3" of rainfall doesn't mean that the definition of the WQV changes. This is important to understand potential limitations of the undersized system.

There are multiple inlet structures specified. The infiltration zone included for the Oyster Pond BMP appears to be a reflection of previous conversations. Traditionally the primary inlet structure is located as far away from the outlet structure as possible to increase the subsurface flow path and limit short circuiting particularly for low flow events.

There should be specifications for all materials incorporated in the design. This includes wetland soil, reservoir stone and 3/8" setting bed stone (not identified).

Hydraulic Modeling

The SGW systems are both modeled dynamically where unit hydrographs for the 0.3" event, the 1-year, 2-year, and 5-year storm events are routed through the system. While residence time varies slightly for each system, in general the modeling indicates that the resident time for the design storms are roughly 10 hours, 20 hours and 30 hours for the 0.3" event, the 1-year event and the 2- year event respectively. Unfortunately this design concept for SGW systems is wrong.

SGW systems are water quality stormwater control measures and are designed statically as described in the 2009 UNHSC design specifications. Fortunately this makes sizing far simpler than the detailed hydraulic routing that has been performed. Unfortunately this will require reconfiguration of both the inlet and outlet controls. The basic concept is that the basin should be statically sized to hold whatever degree of the design storm. The upstream weir elevation should be elevated such that the design in system water elevation will match the invert of the upstream high flow bypass weir control. Obviously adequate freeboard should be designed into the system for flood protection. General guidelines we typically use are a minimum of 2” of freeboard (in this case I think up to 6” may be adequate) and 18-24” of surface ponding elevation. Surface ponding elevations can be flexible however thisis the range that we have tested and design in our own systems. Whatever capacity of the resultant basin geometry becomes the design treatment volume. While this is fundamentally controlled by the upstream weir diversion structure internal bypass contingencies are smart and in general a good approach. From here the design treatment volume is statically drained through the outlet control structure through a simple orifice equation.

For a circular orifice, the equation becomes:

$$Q = Cd(1/4\pi D^2)\sqrt{2gh}$$

Typical values for the coefficient of discharge are:

Sharp orifice: 0.62

Tube: 0.80

The target residence time should be at minimum 24 hours. As we have discussed the longer the residence time the higher potential for nutrient load reductions: a 30 hour drain-down would be preferred. The outlet orifice should have an invert at the top of the subsurface stone lever or up to 4” below the wetland surface. In general the larger the orifice the lower the invert should be within the narrow range described above.

Here the inlet pipe will be elevated above the surface of the wetland cell surface. This will make influent monitoring easier with the exception of the backwater conditions that will be generated during non-design conditions. It may be possible to construct a temporary weir box for the project monitoring period.

We would be happy to discuss this review further over a conference call, or in-person meeting. At this point, while the footprint and conceptual configuration of the system are at an advanced stage, we find various specific design details and the hydraulic routing are not consistent with our specifications, or reflective of the research and findings that have been gathered to date.

Sincerely,

James Houle and Tom Ballestero